Experimental Evaluation of Strengthened Reinforced Concrete T - Joints using GFRP Composite

B. Shivakumara Swamy[1] , S. Vijaya[2] and M. Sekar[3]
[1]*Professor, Civil Engineering Department, Dr. Ambedkar Institute of Technology, Bangalore – 560056*
[2]*Professor, Civil Engineering Department, Dr. Ambedkar Institute of Technology, Bangalore – 56 0056*
[3]*Professor, Structural Engineering Division, Anna University, Chennai-600 025*

Abstract

This study presents an experimental investigation on the behaviour of beam-column joints wrapped with Glass fiber reinforced polymer (GFRP). The behaviours of these joints were investigated under cyclic loading. The test specimens were divided into three groups, the first group consists of beam-column joints designed and detailed as per IS: 456-2000, the second group consists of the columns detailed as per IS: 13920-1993 and the last group strengthened by GFRP. The tests were conducted to investigate the effectiveness of GFRP using Isopthalic polyester resin in strengthening reinforced concrete beam-columns joints. On the basis of test results, it was concluded that GFRP wrapping enhances the energy dissipation capacity. It was observed that the failure mode of beam-columns joints wrapped with of GFRP exhibited sufficient warning time before the ultimate failure.

1. Introduction

The beam-column joints are critical regions in a framed structure because they ensure continuity of a structure and transfer forces from one element to another. The failure of the joint region not only damage the column load paths but also adversely affect the ductility and energy dissipation capacity of the frame as a whole. To improve the performance of these reinforced concrete structures, seismic repair and retrofit methods have been developed in the last three decades. Recently, a new retrofit method involving the use of fibre reinforced plastics (FRP's) has been the centre of attraction. These composites offer advantages over structural steel, reinforced concrete, and timber. Some of the advantages are superior resistance to corrosion, high stiffness-to-weight and strength-to-weight ratio's, and the ability to control the material behaviour by selecting the proper orientation of the fibres [2].All of these features make fibre reinforced composites a highly engineered material suitable for infrastructure applications, in spite of the fact that the cost of FRP is much higher than the cost of conventional construction materials.

The strength of frame connections has been increased by jacketing the joint region in the study performed by Alcor and Jirsa (1993). With the older concrete structures not meeting the criteria recommended by ACI 352R-91, and based on experimental studies, the T-joints designed three decades ago fail at a much lower stress level (Priestley et al. 1997). Therefore rehabilitating beam-column joints represents a feasible approach to mitigate the hazard in existing structures and to provide safety to occupants.

Recent earthquakes tested the vulnerability of existing reinforced concrete (RC) structures to strong ground motions. In many cases, the failure of beam-column joints, especially exterior ones, initiated the collapse of these structures due to the combined effect of bending moment and shear force. Therefore strengthening of concrete beam-column joints is desirable for concrete bridges and other concrete structures built before the 1990's that may have corrosion or seismic problems. The present research focuses on retrofitted techniques to increase the strength based on externally applied GFRP composite.

2. Experimental Programme

To study the behaviour of concrete T-joints, nine test specimens were cast and tested. There were three control specimens designed as per IS: 456-2000 and three specimens designed as per IS: 13920-1993, the remaining three control joints were externally reinforced using composite woven roving GFRP. The selected exterior joints were part of an RC moment resisting frame. The beam and column cross sections selected were 200 mm wide and 200 mm deep to ensure that column failure is avoided and that the effect of shear strength of the joint deficiency is tested. The joint specimens BCT-1a, 1b, and 1c designed and detailed according to IS: 456-2000 is called shear strength deficient joints (control joints) and the joints BCT-2a, 2b, and 2c designed as per IS: 13920-1993 is called ductile detailed specimens. The design compressive strength for the construction of nine specimens was 20 MPa and the measured compressive strength of the concrete was 28 MPa and the yield strength of longitudinal and transverse reinforcement for all the specimens was 415 MPa. Three beam-columns joint BCT-3a, 3b, and 3c are designed as per IS: 456-2000 and strengthened with woven roving GFRP. The GFRP was pasted to the beam bottom and top face and extended 300 mm along the column face, also front and back of beam pasted with 700 mm long woven roving. To prevent the de-bonding of the GFRP at the beam-column junction a U-shaped single layer of GFRP was pasted to the beam and column. The joints were tested in the column vertical position, fixed at both top and bottom ends. A constant axial load of 15 kN was maintained on the column. It represents 1/3 of estimated load of column as per IS: 456-2000. A cyclic load was applied at the beam-tip of the specimen, in steps at an interval of 5 kN till the failure of the beam-column joint.

3. Testing of Beam – Column Joint

The reinforcement details of the beam-column joints are shown schematically in Figure 1 and Figure 2. The specimens were placed on the strong floor with column vertical position. The column ends are prevented from lifting by anchoring them to the floor. The movement of the bottom end of the column at floor level is prevented by the anchorage. Horizontal movement at the top of column was prevented by using two angle sections. The position of connectivity at the top and the bottom of column are assumed as points of contraflexure in the column in the real frame. In addition to the reaction developed in the columns due to the beam tip load, a constant axial load of 15 kN was applied by means of a hydraulic jack. The cyclic load at the tip of the cantilever beam was imposed by means of two hydraulic jacks, one mounted vertically to the frame and the other attached to the strong floor. The hydraulic jacks had a capacity of 10 T (100 kN) and a displacement range of 100 mm. Electrical resistance strain gauges were pasted to the surface of concrete at the joint region to measure strain in concrete surface and two LVDT's, one at the beam tip and the other at the mid span of beam at a distance of 500 mm from the face of the beam to measure the deformation of beam.

3. GFRP Composite

FRP composites are new generation structural materials for civil engineering structures. They are light, strong and corrosion resistant. Due to these advantages they offer great opportunities for the retrofit of existing structures and for constructing high performance structures. In this study for strengthening of beam-columns joints a commercially available woven roving glass fiber reinforced polymer (GFRP), weighing 600 gm/sq.m and Isophthalic polyester resin, which is a general purpose polymer resin was used as an adhesive for pasting GFRP around the joints. To initiate polymerization and hardening, an accelerator and a catalyst were used as prescribed by the supplier. The GFRP and resin were proportioned in the ratio of 1:1 by weight. First the resin was applied on the cleaned surface of joint using brush and then GFRP was pasted. Again the resin was applied over the wrapping to achieve good bonding and they were cured for three days at room temperature. The mechanical properties of the composite were ascertained from tensile test on composite coupons and listed in Table 1.

Table 1: Properties of GFRP Composite

Composite	Tensile strength (MPa)	Strain at rupture	Elastic modulus (GPa)	Poisson's ratio
Bi-directional woven Roving GFRP (Figure 3)	275	0.024	13.75	0.2

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4. Results and Discussions

The load carrying capacities of beam-column joints wrapped with GFRP were obtained and compared to the corresponding unwrapped control beam column joints. Table 2 presents the details regarding the failure modes and maximum moment sustained. The results show that there is substantial gain in the moment carrying capacity for strengthened specimens with GFRP.

Beam tip load versus tip-displacement plots for test specimens BCT-1a and 1b also referred to as hysteretic loops are shown in Figure 4 and Figure 5. The hysteretic loops for test specimens BCT-2a and 2b also are shown in Figure 6 and Figure 7. In the early loading stages of the joint at second cycle minor cracks occurred at the beam-column interface, and a corresponding reduction in the slope of the beam-tip load-displacement relationship was observed. At the beam-tip displacement of 21.6 mm, the splitting crack at the column face widened and extended to the beam. The specimens suffered damage in the form of a wide crack at the beam-column joint and diagonal cracks in the joint. Some flexural cracks also observed in the beam after second cycle. The test was stopped at an average displacement of 61.45 mm, as the load carrying capacity was significantly reduced. The final failure pattern is shown in Figures 8, 9 and 10. The overall behaviour of the joint was brittle and severe strength deterioration.
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4.1 Retrofitted Specimens

The specimen BCT-3a, 3b, and 3c have been strengthened with three layers of woven roving GFRP at the beam column junction, and have been anchored with single layer of GFRP at the joint region. Failure of GFRP initiated with partial de-bonding of the GFRP jacket at the edge of the column when the load reached 30 kN and the displacement is about 55 mm. This may be attributed to the improper anchoring of GFRP to the specimen. This was followed by tensile fracturing of the GFRP on the beam-column interface, with horizontal cracks in the last cycle. At the maximum load of –29.6 kN, splitting of GFRP and widened cracks were observed at the joint region and the maximum displacement observed at this stage was 55.85 mm; the test stopped when the displacement reached 55.85 mm; up to that level the cracks in FRP jacket progressed slowly. Almost identical fracture was observed in the other specimens.

4.2 Strength, Stiffness and Energy Dissipation

To evaluate the effectiveness of the GFRP reinforcement, the strength, the stiffness and the energy dissipation capacity for every load cycle were recorded. The maximum load in both push and pull direction is given in Table 3. The energy dissipated during a loading cycle is calculated as the area enclosed by the load-displacement curve. The capacity of a structure to dissipate energy has a strong influence on its response to an earthquake loading. The cumulative energy dissipated is determined by summing up the energy dissipated in consecutive loops throughout the test. Figure 11 shows the hysteretic envelope curves of all beam-column joint specimens. The strengthened specimens show higher energy dissipation capacity. It is observed that the retrofitted joint with GFRP were capable of dissipating 33% and 9% energy than the energy dissipated by the control joint and ductile joint respectively. Figure 12 shows the cumulative energy dissipated by various beam-column joints.

The stiffness of the joint was evaluated by using the peak-to-peak slope of each cycle (i.e., secant stiffness of the beam-tip load and tip-displacement relationship). Stiffness of various joints tested is shown in Table 3. It is seen that stiffness of strengthened joint BCT-3a, showed higher than the initial stiffness of the control joint (BCT-1a) and (BCT-2a) respectively. After attaining the maximum stiffness, all test specimens showed a general trend in decrease of value till the last displacement applied.

4.3 Failure Mode

The damage of control joints appear at initial level are fine cracks (horizontal and diagonal) without crushing of concrete at the joint region and bending cracks in the beam. The hysteretic loops for retrofitted specimens BCT 3a, 3b and 3c are shown in Figures 13, 14 and 15. The reinforced concrete beam-column joint specimens after failure are as shown in Figures 9 and 10. The joints wrapped with woven roving GFRP (Figures 16, 17 and 18), typically failed by rupture of GFRP composite and de-lamination of wrapping was observed during testing. Small cracks were observed with bending of beam in case of joints wrapped with GFRP. Some popping noises were heard during the failure stages of loading, this sound may be attributed to crushing of GFRP composite. After the failure the confined concrete was found to be not disintegrated. It was observed in rehabilitated beam-columns joints that failure was occurred at the joint region only with the crushing of GFRP composite. Thus by improving the mechanical anchoring, the performance of GFRP can be improved.

5. Conclusions

An experimental program was conducted to evaluate the performance of beam-column joints wrapped with woven roving GFRP and the findings are as follows

1. A brittle failure mode in the form of joint shear failure is the expected failure mode for the deficient joint.
2. Strengthening of beam-column joint using GFRP jacketing found to be an effective system to maintain concrete integrity by confinement, and significantly improves the load carrying capacity of the joint.
3. The beam-column joint confined with GFRP in the joint region showed high strength and large energy dissipation capacity. Stiffness decreases with the increase in number of cycles.
4. The strengthened joint has dissipated 33% higher energy compared to the control joint.
5. De-lamination of GFRP was observed during the last cycle at the joint region even though the anchoring of these layers was improved at the face of the beam and column by providing a transverse layers at those locations.
6. The GFRP reinforced specimens reached their peak load, but as the composite delaminated, this load level could not be sustained. This caused specimen failure at lower loads and corresponding bending moments more than the elements capacity.
7. De-bonding dominates the behaviour of external reinforcement unless proper mechanical anchorages are provided.

6. References

[1] T. El-Amoury, and A. Ghobarah (2002), “Seismic rehabilitation of beam-column joint using GFRP sheets”, Engineering Structures, 24(11), 1397-1407.
[2] Janos Gergely, Chris P. Pantelides and Lawrence D. Reaveley (2000) “Shear strengthening of RC T-Joints using CFRP composites”, Journal of Composites for Construction, 4(2), 56-64.
[3] A. Ghobarah and T. El-Amoury (2005), “Seismic rehabilitation of Deficient Exterior concrete frame joints”, Journal of composites for construction, Vol. 9, No.5, pp 408-416.
[4] S. R. Satish kumar, B. Vijaya Raju and G.S.B. Rajaram (2002) “Hysteretic behaviour of lightly reinforced concrete exterior beam-to-column joint sub-assemblages” Journal of Structural Engineering, Vol.29 No.1, pp 31-37.
[5] Costoas P. Antonopoulos and Thanasis C. Triantafillou (2003), “Experimental Investigation of FRP Strengthened RC Beam-Column joints”, Journal of Composites for Construction, Vol. 7, No.1, pp 39-49
[6] -----IS: 456-2000, Indian Code of practice for plain and reinforced concrete, Bureau of Indian Standards, New Delhi.
[7] -----IS: 13920-1993, “Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces-Code of practice”
[8] George G. Penelis and Andrew J. Kappos, “Earthquake resistant concrete structures”, E & FN SPON, An Imprint of Chapman & Hall, London, 1997.
[9] R. Park and T. Paulay (1975), “Reinforced concrete structures”, John Wiley and Sons, New York.

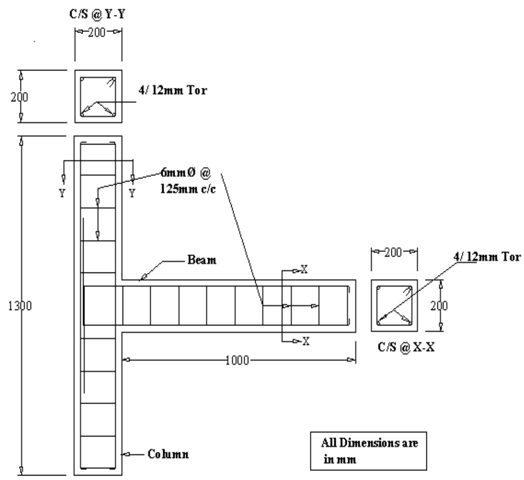

Fig.1: Reinforcement details of beam-column joint (As per IS: 456-2000)

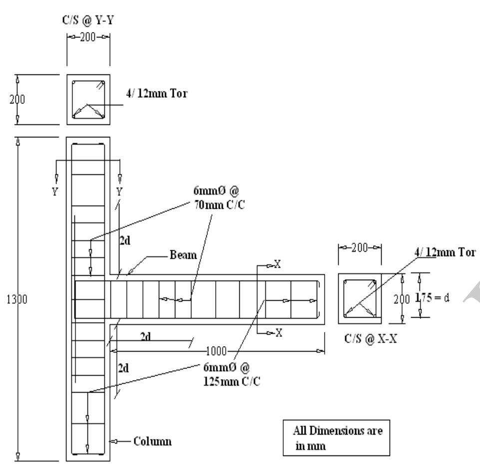

Fig. 2: Reinforcement details of beam-column joint (As per IS: 13920-1993)

Fig. 3: GFRP roving

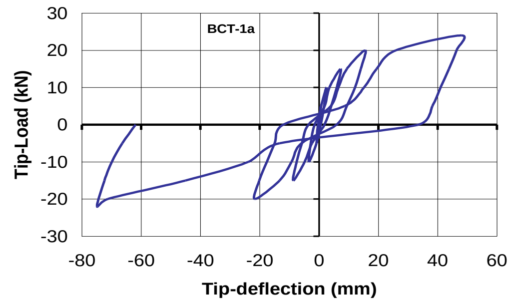

Fig. 4: Hysteretic behaviour of BCT-1a

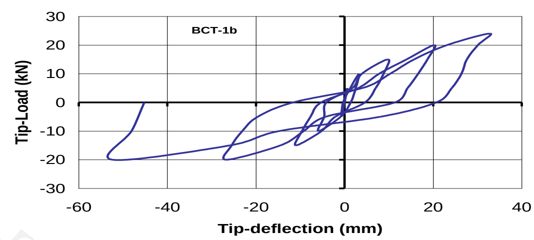

Fig. 5: Hysteretic behaviour of BCT-1b

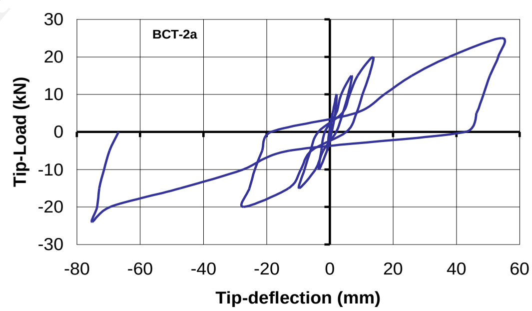

Fig. 5: Hysteretic behaviour of BCT-1b

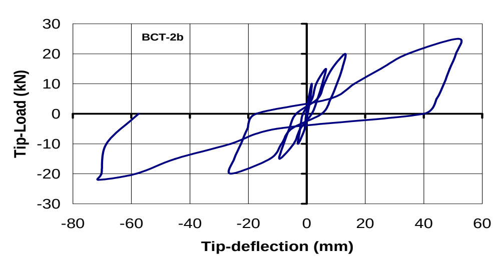

Fig. 7: Hysteretic behaviour of BCT-2b

Fig. 8: Control specimen after test (Failure pattern)

Fig. 9: Flexural failure of IS: 13920 specimens

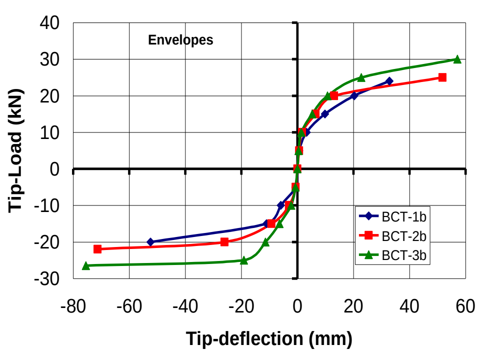

Fig. 10: Hysteretic envelope curves

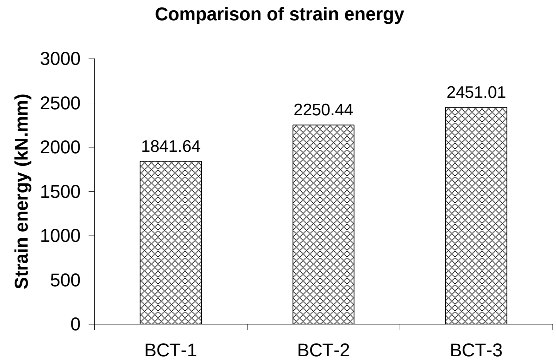

Fig. 11: Comparison of strain energy

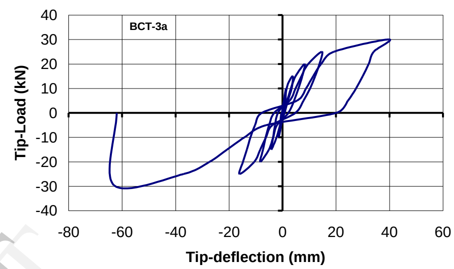

Fig. 12: Hysteretic behaviour of BCT-3a

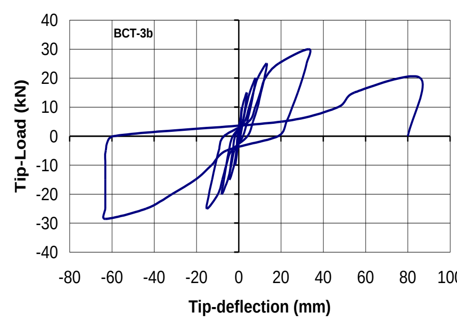

Fig. 13: Hysteretic behaviour of BCT-3a

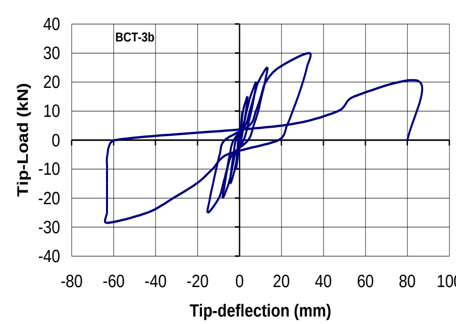

Fig. 14: Hysteretic behaviour of BCT-3b

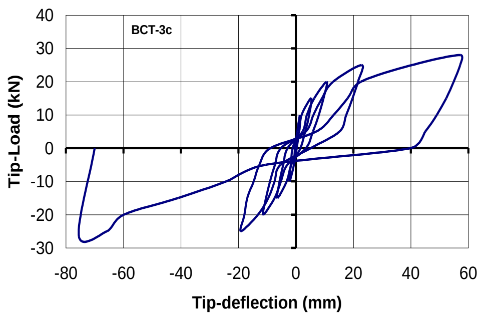

Fig. 15: Hysteretic behaviour of BCT-3c

Fig. 16: Strengthened specimen for testing

Fig. 17: Rupture of GFRP at joint

Fig. 18: De-lamination of GFRP

Table: 2 Experimental Observations

Specimen	Maximum moment sustained		No. of cycles	Mode of failure
	Positive	Negative		
BCT-1a	24	-22	6	Shear
BCT-1b	24	-22	6	Shear
BCT-1c	22	-20	6	Shear
BCT-2a	25	-24	6	Flexure
BCT-2b	25	-22	6	Flexure
BCT-2c	25	-23	6	Flexure
BCT-3a	30	-29.6	7	Rupture of GFRP
BCT-3b	30	-26.5	7	Rupture of GFRP
BCT-3c	28	-26.5	7	Rupture of GFRP